APPENDIX C
SLOPE STABILITY ANALYSIS

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SLOPE STABILITY ANALYSIS
BASIS OF DESIGN REPORT
JORGENSEN FORGE EARLY ACTION AREA

Prepared for

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On behalf of

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March 2013

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LIST OF ACRONYMS AND ABBREVIATIONS

AOC	Administrative Settlement Agreement and Order on Consent for Removal Action Implementation
Basis of Design Report	Basis of Design Report
CERCLA	Comprehensive Environmental Response, Compensation, and Liability Act
DAR	Design Analysis Report
EAA	Early Action Area
EMJ	Earle M. Jorgensen
EPA	U.S. Environmental Protection Agency
Facility	Jorgensen Forge facility
g	acceleration due to gravity
H:V	Horizontal to Vertical
Jorgensen Forge	Jorgensen Forge Corporation
LDW	Lower Duwamish Waterway
MLLW	mean lower low water
NTCRA	non-time-critical removal action
Owner	EMJ and Jorgensen Forge
pcf	per cubic foot
PGA	peak ground acceleration
SM	Silty sand
SOW	Statement of Work
SP	poorly graded sand
USACE	U.S. Army Corps of Engineers
WSDOT	Washington State Department of Transportation

EMJ015140

1 GENERAL

This Slope Stability Report was prepared on behalf of Earle M. Jorgensen Company (EMJ) and Jorgensen Forge Corporation (Jorgensen Forge; herein referred to collectively as the Owner) pursuant to the Administrative Settlement Agreement and Order on Consent for Removal Action Implementation (AOC; U.S. Environmental Protection Agency [EPA] Region 10 Comprehensive Environmental Response, Compensation, and Liability Act [CERCLA] Docket No. 10-2012-0032) and attached Statement of Work (SOW). This Slope Stability Report is an appendix to the Basis of Design Report (BODR) Final Design submittal for the cleanup of contaminated sediments and associated bank soils in a portion of the Lower Duwamish Waterway (LDW) Superfund Site adjacent to the Jorgensen Forge facility (Facility) located in Tukwila, King County, Washington (see Figure 1 of the BODR; Jorgensen Forge Early Action Area [EAA]). The cleanup will be conducted as a non-time-critical removal action (NTCRA) in accordance with EPA's selected cleanup alternative documented in the *Action Memorandum for a Non-Time-Critical Removal Action at the Jorgensen Forge Early Action Area of the Lower Duwamish Waterway Superfund Site in Seattle, Washington* (Action Memo; EPA 2011) and detailed in the *Final Engineering Evaluation/Cost Analysis [EE/CA]– Jorgensen Forge Facility, 8531 East Marginal Way South, Seattle, Washington* (Anchor QEA 2011). The Jorgensen Forge EAA is located near River Miles (RMs) 3.6 to 3.7 on the east bank of the LDW.

The evaluations presented in this appendix indicate that the proposed engineered slopes meet the required performance standards for short-term and long-term stability. Slope stability analysis indicates that the proposed design meets the long-term slope stability performance standard of 1.3 of the Washington State Department of Transportation (WSDOT) *Geotechnical Design Manual* with an estimated factor of safety of 1.5 (WSDOT 2011). Consistent with the work of others (Integral 2007), post-shoreline containment stability was evaluated for a ground motion, which represented a 100-year earthquake. For this pseudostatic analysis, the minimum factor of safety of 1.06 meets the required performance standard of 1.05 (WSDOT 2011).

For short-term stability of the dredged slope, the estimated factor of safety is 1.2 prior to shoreline containment placement.

1.1 General

Slope stability evaluations were performed with SLIDE6.0, a commonly used geotechnical engineering software program which uses a trial and error algorithm to determine the critical slip surface, which is the plane that represents the lowest factor of safety for a user-specified soil profile.

The lowest factor of safety determined from limit equilibrium modeling is compared to performance standards. Performance standards used in this appendix are based off of the WSDOT *Geotechnical Design Manual* and the U.S. Army Corps of Engineers (USACE) Slope Stability Guide (USACE 2003). For general analysis of permanent slopes involving cuts and fills not adjacent to structures, a minimum factor of safety of 1.3 is appropriate. For seismic analysis, the minimum required factor of safety is 1.05. For temporary cuts to embankments not supporting structures, the safety of workers is the primary concern. Therefore, a safety factor of approximately 1:1 should be met.

1.2 Inputs

Geologic cross sections were interpreted using on-site boring logs, and supplemented by geotechnical data from nearby sites. Soil parameters for the model were determined using boring log data, which included soil observations and classifications, Standard Penetration Resistance, and laboratory results when available. Using empirically published correlations and available geotechnical data, Table 1 summarizes the input parameters for the stability evaluation. Figure 1 presents the generalized soil profile.

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Table 1
Summary of SLIDE Input Parameters

Soil Layer	Unit Weight (pcf)	Saturated Unit Weight (pcf)	Friction Angle (Degrees)
1 - Loose Sand - SP	115	118	30
2 - Fill, cobbles, sand	124	137	35
3 - Dense Sand - SP	121	122	42
4 - Very Loose Sand - SM	115	118	28
5 - Shoreline Containment Material	125	130	36
6 - Loose Sand Backfill	115	116	28

Notes:
pcf = per cubic foot
SM = Silty sand
SP = Poorly graded sand

Slope stability was evaluated where deposits of loose soils were present, and where existing grades were steepest. This is represented by the cross section presented in Figure 1. The steepest portion of this slope is approximately 1H:1V (Horizontal to Vertical), approximately 10feet high, and in an over-steepened condition, as indicated by observed erosion. Below the over-steepened portion, the slope is flatter than 1.5H:1V, eventually flattening to approximately 6H:1V near the channel. The underlying loose sand layer is approximately 12 feet thick and extends to the proposed toe of the designed shoreline containment slope.

Consistent with the slope stability evaluations provided by others in Appendix E of the Slip 4 Early Action 100% Design Submittal (Integral 2007):

- Slope stability evaluations were performed using Spencer's Method of Slices.
- The minimum critical failure depth was set to 5 feet.
- Longer–term slope stability evaluations were performed considering a low tide of 0 mean lower low water (MLLW) and a high tide of 12 MLLW.

1.3 Results

1.3.1 Existing Conditions

Slope stability modeling indicated that the minimum factor of safety of the existing slope is near 1.0. The critical slip surface shown on Figure 2 occurs in the over-steepened portion of the slope.

Inspection of the contours presented in Figure 2shows that at a distance of 15 feet from the crest of the slope, the factor of safety is greater than 1.5. This indicates that deformations would likely be limited to shallow slope failures, and is consistent with observations at the site.

1.3.2 Post-dredge Temporary Slope

The short-term stability of the temporary post-dredge slope was evaluated assuming a 2H:1V cut back slope, and a dredge cut consistent with the proposed remedial design. Figure 3 indicates a minimum factor of safety of 1.2 for the temporary construction slope. As shown in Figure 3, the critical surface occurs in the underlying loose sandy layer.

1.3.3 Post-shoreline Containment

Consistent with the proposed design, long-term stability of the slope was evaluated assuming a 4-foot-thick shoreline containment placed on the slope after excavation of the existing soils. Stability analyses shown in Figure 4 indicate a minimum factor of safety of 1.5.

1.3.4 Seismic

Consistent with the work of others (Integral 2007), a peak ground acceleration (PGA) of 0.15 g (acceleration due to gravity) is anticipated to represent the ground motion experienced during a 100-year earthquake. The resultant horizontal earthquake loading coefficient of 0.075g was chosen to represent the acceleration that the soil mass will respond to. The acceleration is then multiplied by the weight of the mobilized soil layers to estimate the de-stabilizing force of the earthquake. As shown in Figure 5, the minimum factor of safety for the seismic condition is 1.1.

1.4 Liquefaction Potential

Soil conditions and densities were evaluated to determine the likelihood of liquefaction during a design-level seismic event. The potential for liquefaction was evaluated for the design level of seismic activity, with a recurrence interval of roughly 100 years, a peak ground acceleration of 0.15g, and a magnitude of 6.5. Factors of safety were computed based on SPT blow counts, percent fines, and depth of layers using the method of Seed et al. (1985).

Our analysis indicates that during a design level event, the loose sand layer presented in Figure 1 would be susceptible to liquefaction.

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2 CONCLUSIONS

Table 2 summarizes the factors of safety and performance standards for each loading condition. The proposed design meets the required short-term and long-term performance standards, and will likely improve the stability of the bank by flatting the slopes, and increasing the strength of the surface soils by construction of a shoreline containment layer which has a higher strength than current sandy layer.

While the factor of safety for the temporary construction slope is low, the overall stability of the embankment should be improved from the existing condition. The temporary construction slope stability evaluation assumes that the entire slope will be dredged prior to slope containment, and that the exposed surface will be the loose sand unit. If shallow slope failures occur during dredging, excavation of smaller sections of the slope and immediately placing shoreline containment will improve temporary slope stability.

Table 2
Summary of Minimum Factors of Safety and Performance Criteria

Scenario	Loading Condition (Long-term or Short-term)	Factor of Safety	Performance Standard
Existing Conditions	Long-term	1	Not Applicable
Post-dredge, Temporary Construction	Short-term	1.2	1.2
Post-shoreline Containment	Long-term	1.5	1.3
Seismic	Short-term	1.1	1.05

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3 REFERENCES

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H. B. Seed, K. Tokimatsu, L. F. Harder, and R. M. Chung, 1985. Influence of SPT Procedures in Soil Liquefaction Resistance Evaluation. *ASCE Journal of Geotechnical Engineering*, Vol.111, No.12, December 1985.

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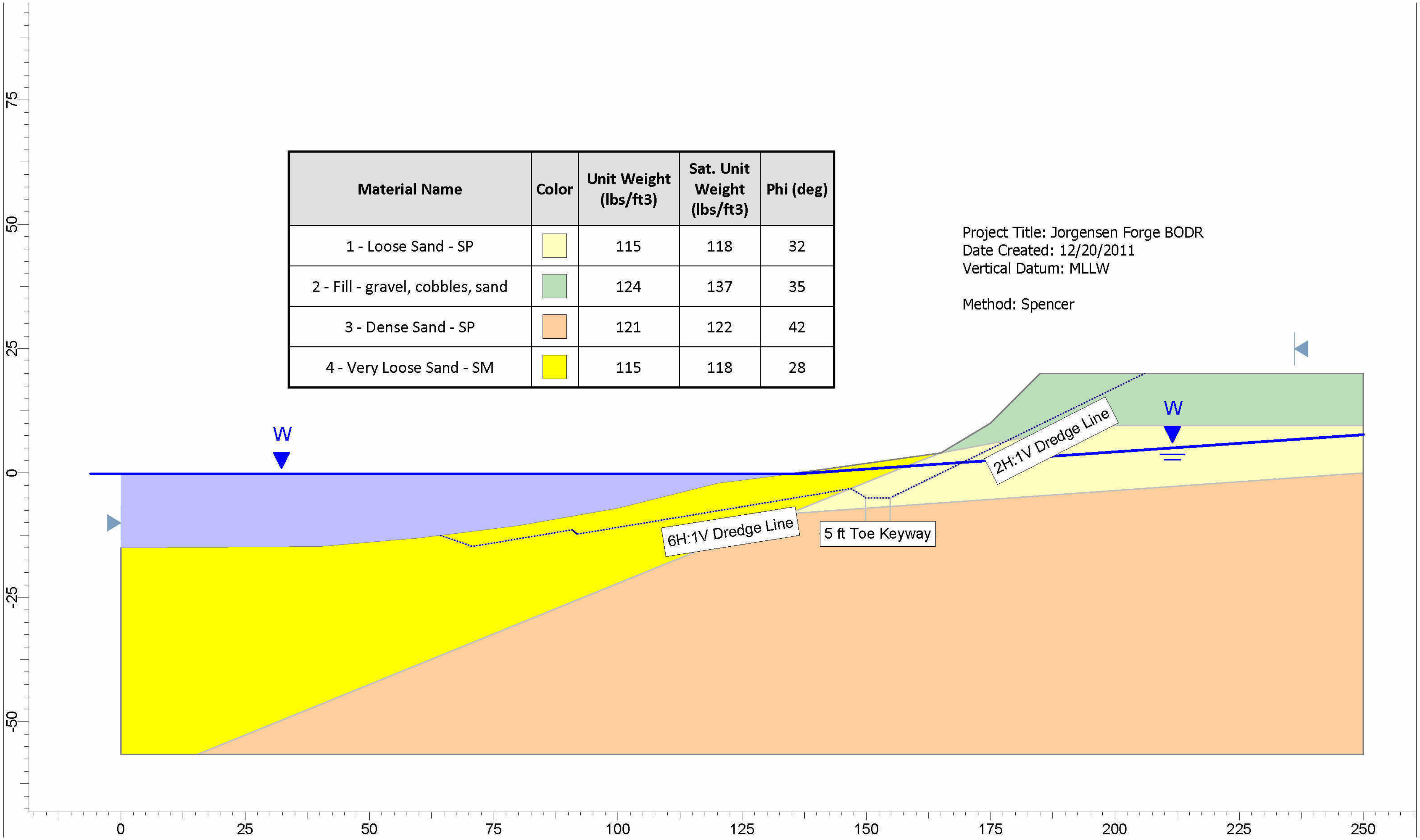
FIGURES



Material Name	Color	Unit Weight (lbs/ft3)	Sat. Unit Weight (lbs/ft3)	Phi (deg)
1 - Loose Sand - SP		115	118	32
2 - Fill - gravel, cobbles, sand		124	137	35
3 - Dense Sand - SP		121	122	42
4 - Very Loose Sand - SM		115	118	28

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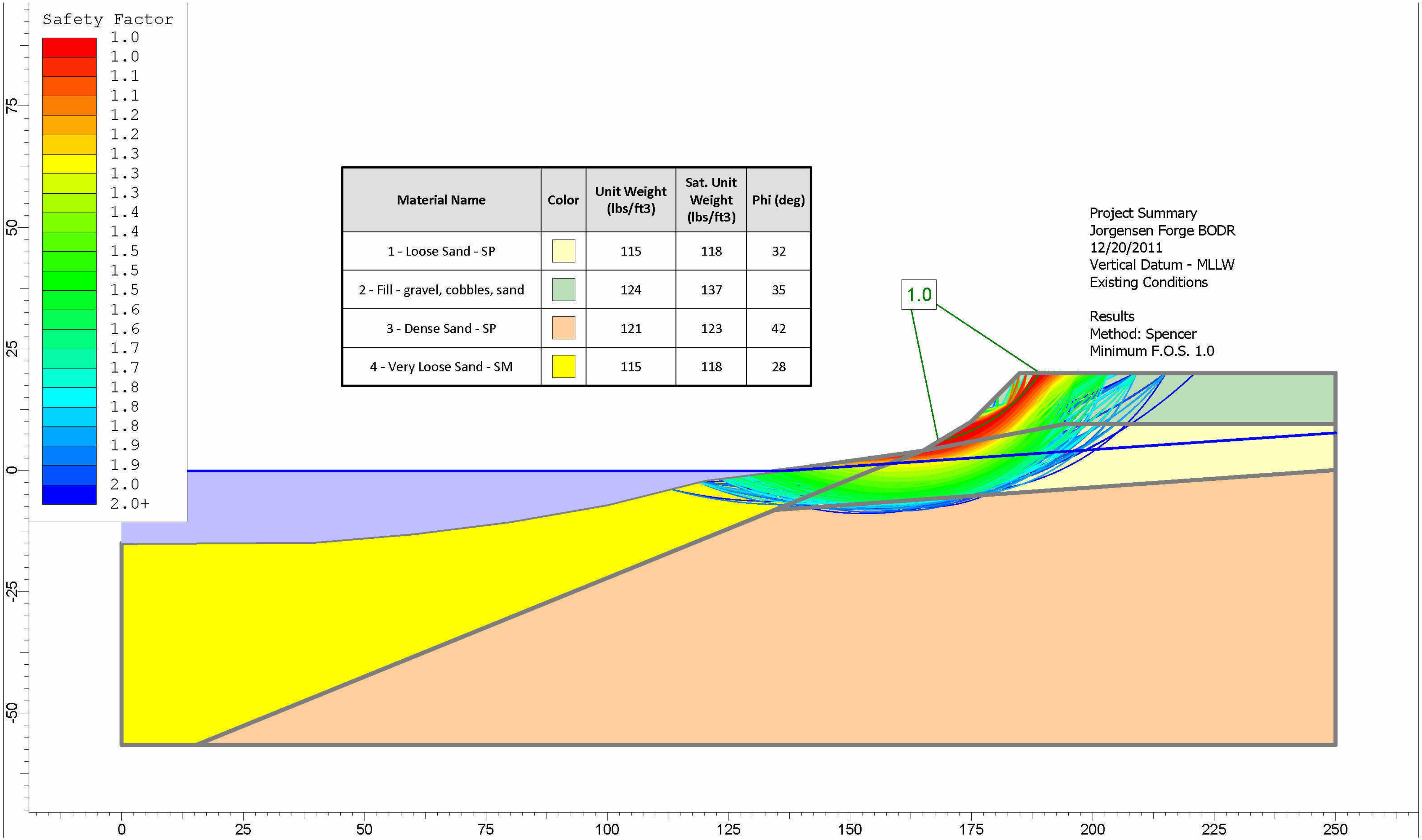
Figure 1
Existing Conditions
Slope Stability Analysis
Jorgensen Forge Facility



Material Name	Color	Unit Weight (lbs/ft3)	Sat. Unit Weight (lbs/ft3)	Phi (deg)
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3 - Dense Sand - SP		121	123	42
4 - Very Loose Sand - SM		115	118	28

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Figure 2
Geologic Cross Section and Existing Conditions
Slope Stability Analysis
Jorgensen Forge Facility

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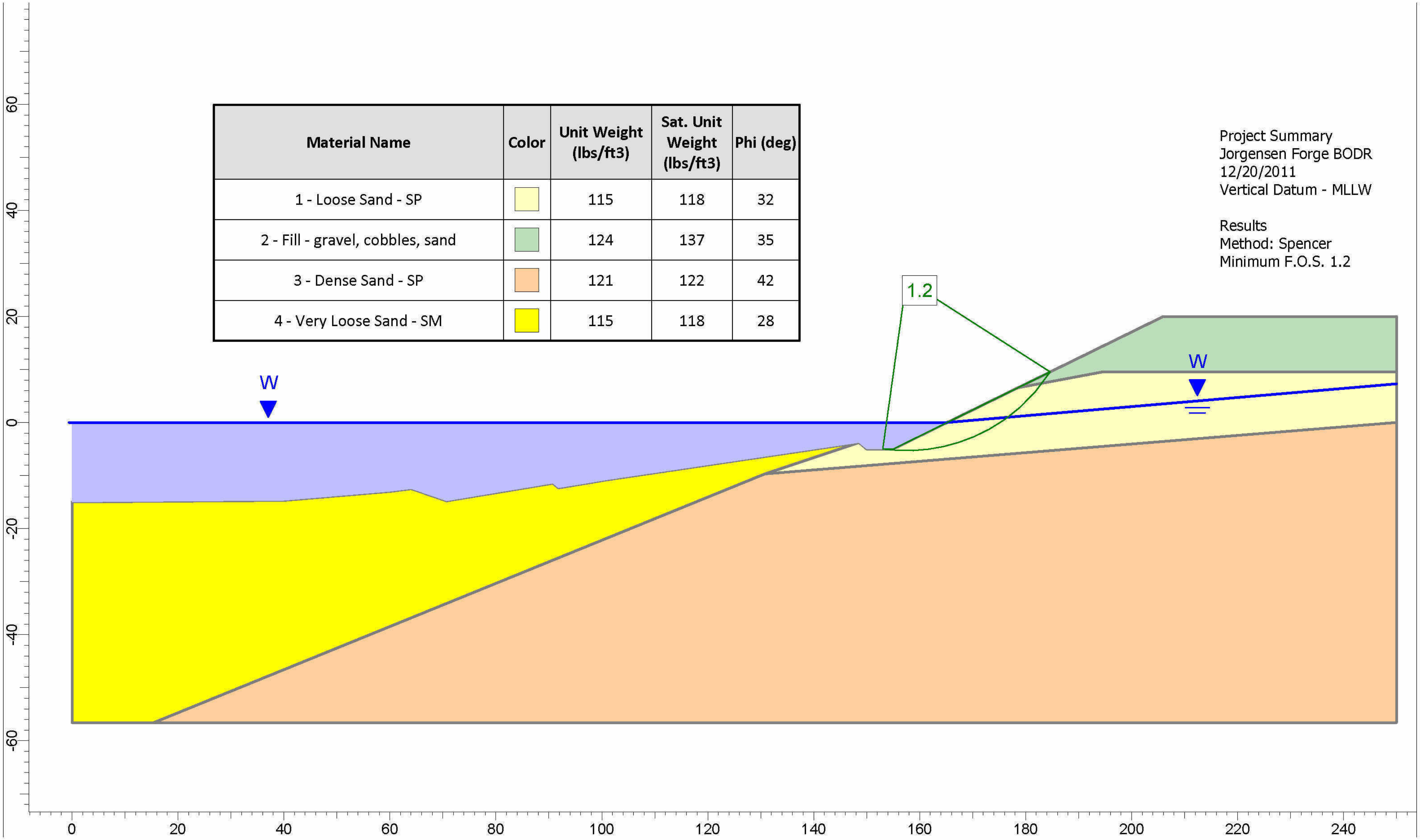

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2 - Fill - gravel, cobbles, sand		124	137	35
3 - Dense Sand - SP		121	122	42
4 - Very Loose Sand - SM		115	118	28

Project Summary
Jorgensen Forge BODR
12/20/2011
Vertical Datum - MLLW

Results
Method: Spencer
Minimum F.O.S. 1.2

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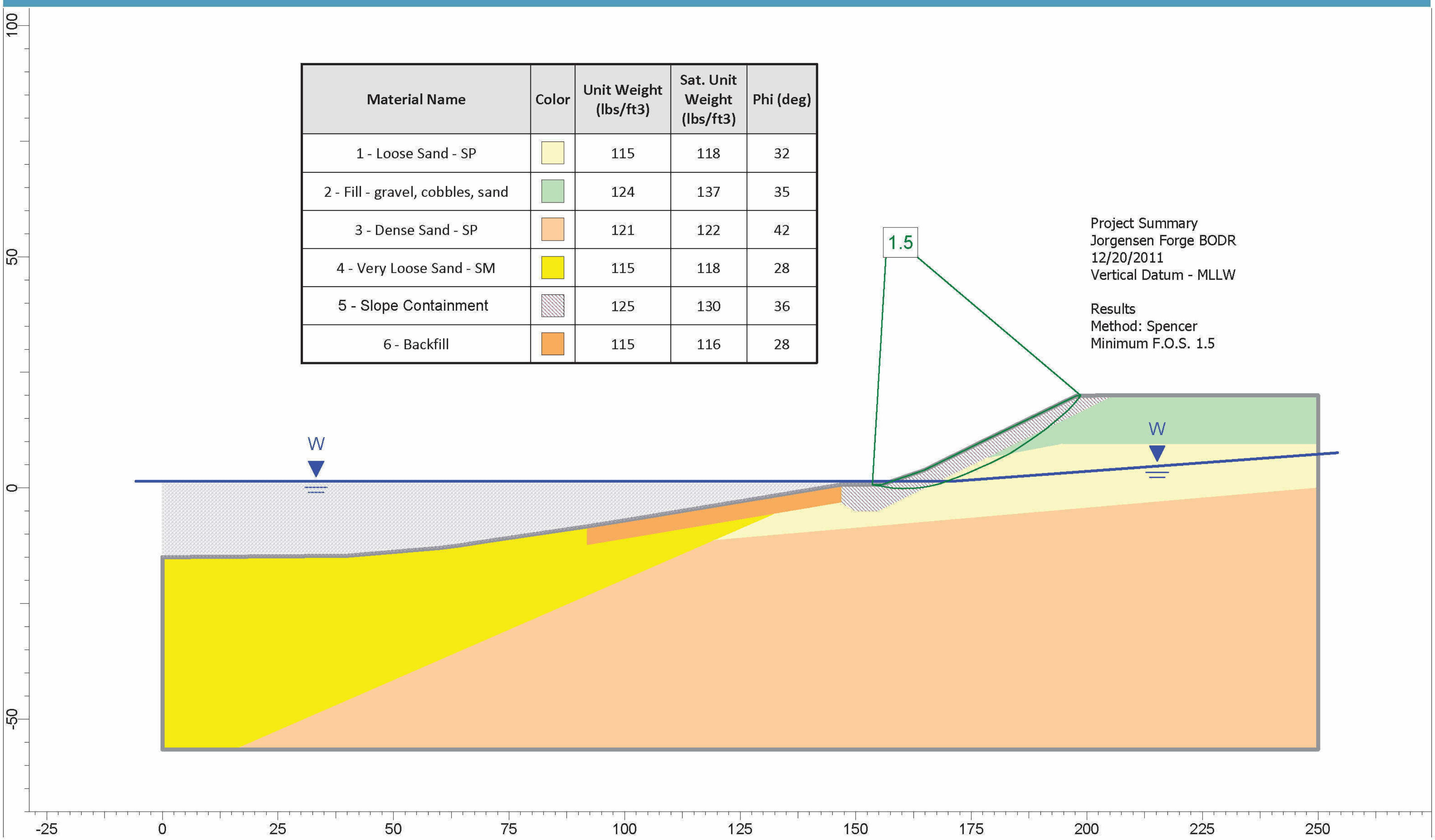
Figure 3
Post-dredge Temporary Slope
Slope Stability Analysis
Jorgensen Forge Facility



Material Name	Color	Unit Weight (lbs/ft3)	Sat. Unit Weight (lbs/ft3)	Phi (deg)
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3 - Dense Sand - SP		121	122	42
4 - Very Loose Sand - SM		115	118	28
5 - Slope Containment		125	130	36
6 - Backfill		115	116	28

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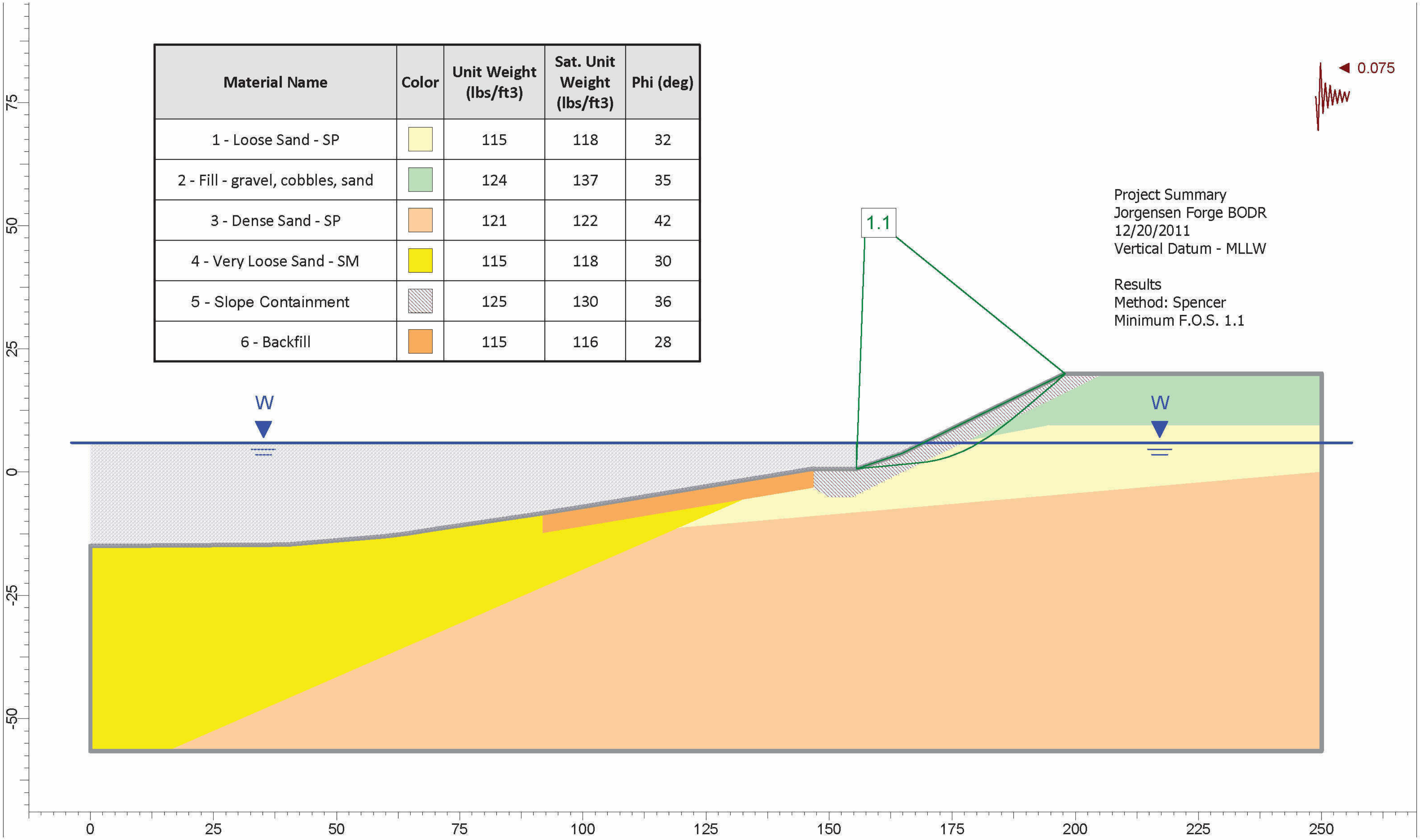
Figure 4
Post Slope Containment : Long Term
Slope Stability Analysis
Jorgensen Forge Facility



Material Name	Color	Unit Weight (lbs/ft3)	Sat. Unit Weight (lbs/ft3)	Phi (deg)
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5 - Slope Containment		125	130	36
6 - Backfill		115	116	28

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Figure 5
Post Slope Containment : Seismic
Slope Stability Analysis
Jorgensen Forge Facility